7 m diameter steel sewer being installed in wet conditions.

CHAPTER 3 Hydrology

Introduction

The hydrologic cycle is a continuous process whereby water is transported from ocean and land surfaces to the atmosphere from which it falls again in the form of precipitation. There are many inter-related phenomena involved in this process and these are often depicted in a simplistic form as shown in Figure 3.1. Different specialist interests, such as meteorologists, oceanographers or agronomists, focus on different components of the cycle, but from the point of view of the drainage engineer, the relevant part of the cycle can be represented in idealistic fashion by the block diagram of Figure 3.2.

The effect of urbanization on the environment is to complicate that part of the hydrologic cycle which is affected by the modification of natural drainage paths, impounding of water, diversion of storm water and the implementation of storm water management techniques.

The objective of this chapter is to introduce the drainage engineer to different methods for estimating those components of the hydrologic cycle which affect design decisions—from precipitation to runoff. Emphasis is placed on the description of alternative methods for analyzing or simulating the rainfall-runoff process, particularly where these apply to computer models. This should help the user of these models in determining appropriate data and interpreting the results thereby lessening the “black box” impression with which users are often faced.

Inevitably it is necessary to describe many of these processes in mathematical terms. Every effort has been made to keep the presentation simple but some fundamental knowledge of hydrology has been assumed.

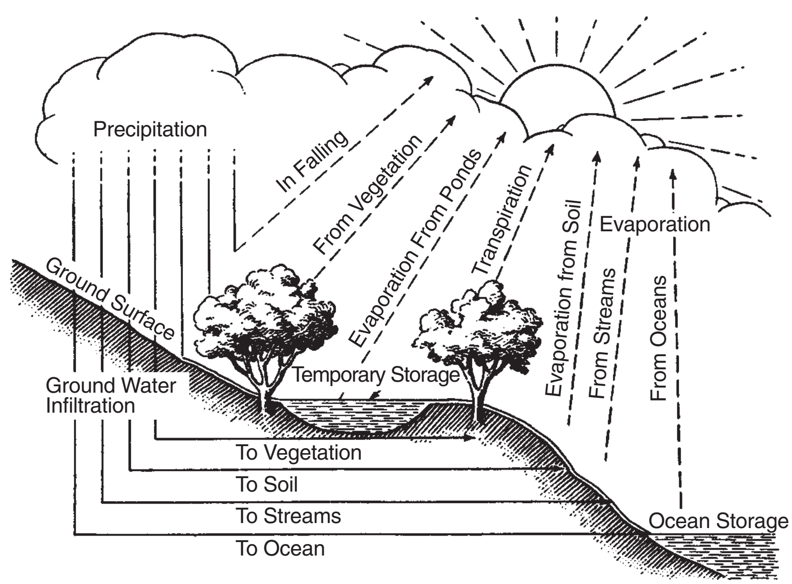

Figure 3.1 Hydrologic cycle - where water comes from and where it goes. From M. G. Spangler’s “Soil Engineering” [1]

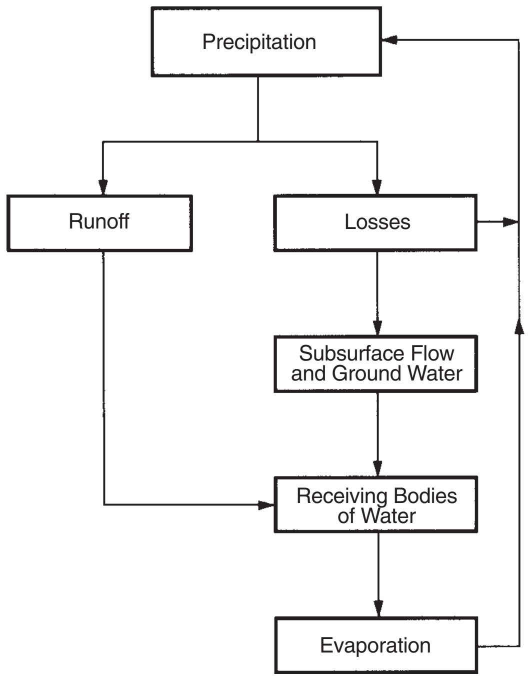

Figure 3.2 Block diagram—Hydrologic Cycle

ESTIMATION OF RAINFALL

The initial data required for drainage design is a description of the rainfall. In most cases this will be a single event storm, i.e., a period of significant and continuous rainfall preceeded and followed by a reasonable length of time during which no rainfall occurs. Continuous rainfall records extending many days or weeks may sometimes be used for the simulation of a system, particularly where the quality rather than the quantity of runoff water is of concern.

The rainfall event may be either historic, taken from recorded events or idealized. The main parameters of interest are the total amount (or depth) of precipitation (P_{tot}), the duration of the storm (t_d) and the distribution of the rainfall intensity (i) throughout the storm event. The frequency of occurrence (N) of a storm is usually expressed in years and is an estimate based on statistical records of the long-term average time interval which is expected to elapse between successive occurrences of two storms of a particular severity (e.g., depth P_{tot} in a given time t_d). The word "expected" is emphasized because there is absolutely no certainty that after a 25-year storm has occurred, a storm of equal or greater severity will not occur for another 25 years. This fact, while statistically true, is often difficult to convey to residents of an area.

Rainfall Intensity-Duration Frequency Curves

Rainfall intensity-duration frequency curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a certain return frequency, from a series of storms. These curves are then said to represent storms of a specific return frequency.

The intensity, or the rate of rainfall is usually expressed in depth per unit time with the highest intensities occurring over short time intervals and progressively decreasing as the time intervals increase. The greater intensity of the storm, the lesser their recurrence frequency; thus the highest intensity for a specific duration for N years of records is called the N year storm, with a frequency of once in N years.

The curves may be in the graphical form as the example shown in Figure 3.3, or may be represented by individual equations that express the time intensity relationships for specific frequencies, in the form:

$$i = \frac{a}{(t + c)^b}$$

where i = intensity (mm/hr)

t = time in minutes

a, b, c = constants developed for each IDF curve

The fitting of rainfall data to the equation may be obtained by either graphical or least square methods.[2]

It should be noted that the IDF curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of N frequency.

The rainfall intensity-duration curves are readily available from governmental agencies, local municipalities and towns, and as such are widely used in the designing of storm drainage facilities and flood flow analysis.

Rainfall Hyetographs

The previous section discussed the dependence of the average rainfall intensity of a storm on various factors. Of great importance from historical rainfall events is the way in which the precipitation is distributed in time over the duration of the storm. This can be described using a rainfall hyetograph which is a graphical representation of the variation of rainfall intensity with time. Rainfall hyetographs can be obtained (usually in tabular rather than graphical form) from weather stations which have suitable records of historical rainfall events. Figure 3.4 shows a typical example.

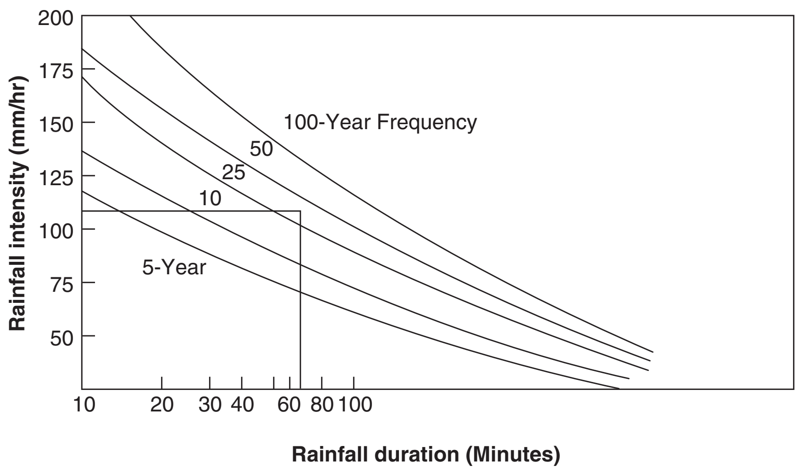

Figure 3.3 Rainfall intensities for various storm frequencies vs. rainfall duration

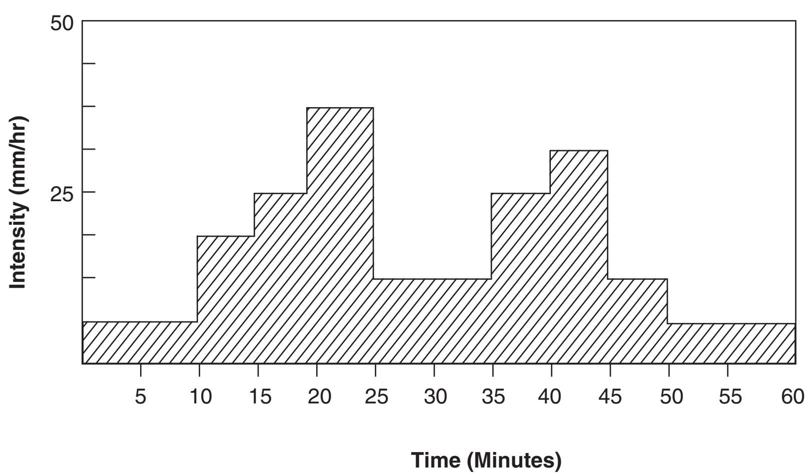

Figure 3.4 Rainfall hyetograph

Conventionally, rainfall intensity is plotted in the form of a bar graph. It is thus implicitly assumed that the rainfall intensity remains constant over the timestep used to describe the hyetograph. Obviously this approximation becomes a truer representation of reality as the timestep gets smaller. However, very small timesteps may require very large amounts of data to represent a storm and can increase the computational cost of simulation considerably. At the other extreme, it is essential that the timestep not be too large, especially for short duration events or very small catchments, otherwise peak values of both rainfall and runoff can be "smeared" with consequent loss of accuracy. When using a computer model this point should be kept in mind since it is usual to employ the same timestep for both the description of the rainfall hyetograph and the computation of the runoff hyetograph. Choice of timestep is therefore influenced by:

a) accuracy of rainfall-runoff representation,
b) discretization of the available data,
c) size of the watershed, and
d) computational storage and cost.

Synthetic Rainfall Hyetographs

An artificial or idealized hyetograph may be required for a number of reasons, two of which are noted here.

a) The historic rainfall data may not be available for the location or the return frequency desired.
b) It may be desirable to standardize the design storm to be used within a region in order that comparisons of results from various studies may be made.

The time distribution of the selected design hyetograph will significantly affect the timing and magnitude of the peak runoff. Care should therefore be taken in selecting a design storm to ensure that it is representative of the rainfall patterns in the area under study. In many cases, depending upon the size of the watershed and degree of urbanization, it may be necessary to use several different rainfall hyetographs to determine the sensitivity of the results to the different design storms. For example, when runoff from pervious areas is significant, it will be found that late peaking storms produce higher peak runoff than early peaking storms of the same total depth as the latter tend to be reduced in severity by the initially high infiltration capacity of the ground.

Selection of the storm duration will also influence the hydrograph characteristics. The Soil Conservation Service Handbook[3] recommends that a six hour storm duration be used for watersheds with a time of concentration less than or equal to six hours. For watersheds where the time of concentration exceeds six hours the storm duration should equal the time of concentration.

A number of different synthetic hyetographs are described in the following sections. These include:

a) uniform rainfall as in the rational method,
b) the Chicago hyetograph,
c) the SCS design storms,
d) Huff's storm distribution patterns.

Uniform Rainfall

The simplest possible design storm is to assume that the intensity is uniformly distributed throughout the storm duration. Thus

$$i = i_{ave} = \frac{P_{tot}}{t_d}$$

This simplified approximation is used in the rational method with the further assumption that the storm duration is equal to the time of concentration of the catchment. (see Figure 3.5). Use of a rectangular rainfall distribution is seldom justified or acceptable nowadays, except for first cut or "back-of-the-envelope" estimates. It can, however, have some use in explaining or visualizing rainfall runoff processes since any hyetograph may be considered as a series of such uniform, short duration pulses of rainfall.

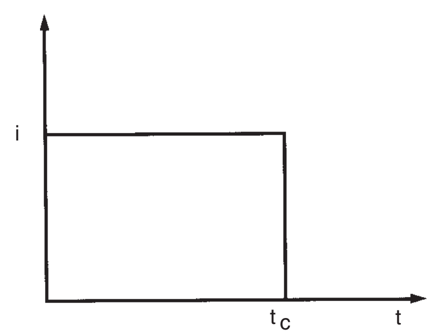

Figure 3.5 Uniform rainfall

The Chicago Hyetograph

The Chicago hyetograph[4] is assumed to have a time distribution such that if a series of ever increasing "time-slices" were analyzed around the peak rainfall, the average intensity for each "slice" would lie on a single curve of the IDF diagram. It, therefore, implies that the Chicago design storm displays statistical properties which are consistent with the statistics of the IDF curve. The synthesis of the Chicago hyetograph, therefore, starts with the parameters of an IDF curve together with a parameter (r) which defines the fraction of the storm duration which occurs before the peak rainfall intensity. The value of r is derived from the analysis of actual rainfall events and is generally in the range of 0.3-0.5.

The continuous curves of the hyetograph in Figure 3.6 can be computed in terms of the times before (t_b) or after (t_a) the peak intensity by the two equations below.

a) After the peak

$$i_a = \frac{a\left[(1 - b)\dfrac{t_a}{1 - r} + c\right]}{\left(\dfrac{t_a}{1 - r} + c\right)^{1 + b}}$$

b) Before the peak

$$i_b = \frac{a\left[(1 - b)\dfrac{t_b}{r} + c\right]}{\left(\dfrac{t_b}{r} + c\right)^{1 + b}}$$

where: t_a = time after peak
t_b = time before peak
r = ratio of time before the peak occurs to the total duration time

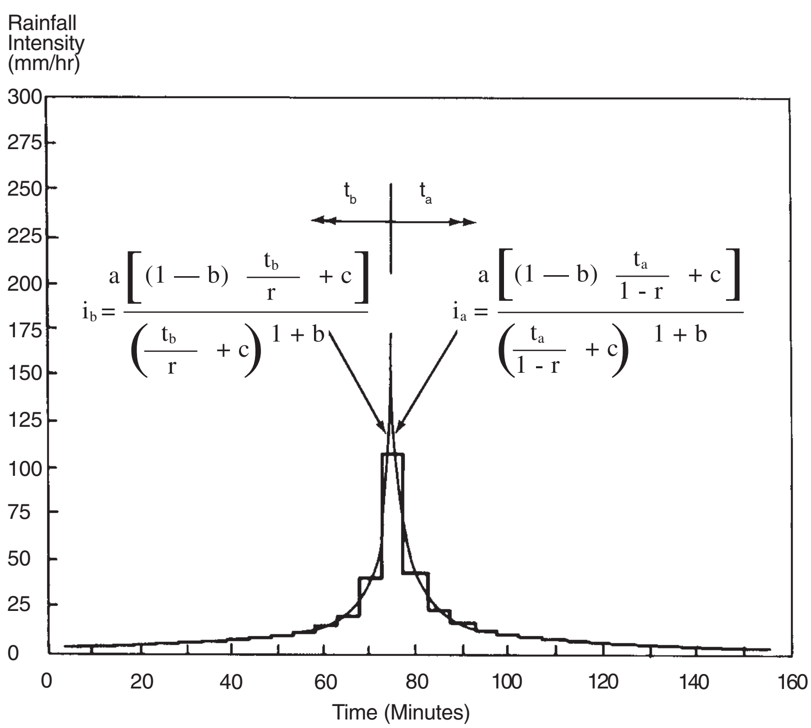

Figure 3.6 Chicago hyetograph

The Chicago storm is commonly used for small to medium watersheds (0.25 km^2 to 25 km^2) for both rural or urbanized conditions. Typical storm durations are in the range of 1.0 to 4.0 hours. It has been found that peak runoff flows computed using a Chicago design storm are higher than those obtained using other synthetic or historic storms. This is due to the fact that the Chicago storm attempts to model the statistics of a large collection of real storms and thus tends to present an unrealistically extreme distribution. Another point to note is that the resultant peak runoff may exhibit some sensitivity to the time step used; very small timesteps giving rise to slightly more peaked runoff hydrographs.

The Huff Rainfall Distribution Curves

Huff[5] analyzed the significant storms in 11 years of rainfall data recorded by the State of Illinois. The data was represented in non-dimensional form by expressing the accumulated depth of precipitation P_t (i.e., at time t after the start of rainfall) as a fraction of the total storm depth P_{tot} and plotting this ratio as a function of a non-dimensional time t/t_d.

The storms were grouped into four categories depending on whether the peak rainfall intensity fell in the 1st, 2nd, 3rd or 4th quarter (or quartile) of the storm duration. In each category a family of curves was developed representing values exceeded in 90%, 80%, 70%, etc., of the storm events. Thus the average of all the storm events in a particular category (e.g., 1st quartile) is represented by the 50% exceedence curve. Table 3.1 shows the dimensionless coefficients for each quartile expressed at intervals of 5% of t_d.

The first quartile curve is generally associated with relatively short duration storms in which 62% of the precipitation depth occurs in the first quarter of the storm duration. The fourth quartile curve is normally used

Table 3.1 Dimensionless Huff storm coefficients

	P_t/P_{tot} for quartile			
t/t_d	1	2	3	4
0.00	0.000	0.000	0.000	0.000
0.05	0.063	0.015	0.020	0.020
0.10	0.178	0.031	0.040	0.040
0.15	0.333	0.070	0.072	0.055
0.20	0.500	0.125	0.100	0.070
0.25	0.620	0.208	0.122	0.085
0.30	0.705	0.305	0.140	0.100
0.35	0.760	0.420	0.155	0.115
0.40	0.798	0.525	0.180	0.135
0.45	0.830	0.630	0.215	0.155
0.50	0.855	0.725	0.280	0.185
0.55	0.880	0.805	0.395	0.215
0.60	0.898	0.860	0.535	0.245
0.65	0.915	0.900	0.690	0.290
0.70	0.930	0.930	0.790	0.350
0.75	0.944	0.948	0.875	0.435
0.80	0.958	0.962	0.935	0.545
0.85	0.971	0.974	0.965	0.740
0.90	0.983	0.985	0.985	0.920
0.95	0.994	0.993	0.995	0.975
1.00	1.000	1.000	1.000	1.000

for longer duration storms in which the rainfall is more evenly distributed over the duration t_d and is often dominated by a series of rain showers or steady rain or a combination of both. The third quartile has been found to be suitable for storms on the Pacific seaboard.

The study area and storm duration for which the distributions were developed vary considerably, with t_d varying from 3 to 48 hours and the drainage basin area ranging from 25 to 1000 km.[2] The distributions are most applicable to midwestern regions of North America and regions of similar rainfall climatology and physiography.

To use the Huff distribution the user need only specify the total depth of rainfall P_{tot} the duration t_d and the desired quartile. The curve can then be scaled up to a dimensional mass curve and the intensities obtained by discretizing the mass curve for the specified timestep, t.

SCS Storm Distributions

The U.S. Soil Conservation Service design storm was developed for various storm types, storm durations and regions in the United States.[3] The storm duration was initially selected to be 6 hours. Durations of up to 48 hours have, however, been developed. The rainfall distribution varies, based on duration and location. The 6, 12 and 24 hour distributions for the SCS Type II storm are given in Table 3.2. This distribution is used in all regions of the United States and Canada with the exception of the Pacific coast.

The design storms were initially developed for large (25km^2) rural basins. However, both the longer duration (6 to 48 hour) and shorter 1 hour thunderstorm distributions have been used in urban areas and for smaller areas.

Table 3.2 SCS Type II rainfall distribution for 3h, 6h, 12h and 24h durations

3 Hour			6 Hour			12 Hour			24 Hour		
Time end'g	F_{inc} (%)	F_{cum} (%)	Time end'g	F_{inc} (%)	F_{cum} (%)	Time end'g	F_{inc} (%)	F_{cum} (%)	Time end'g	F_{inc} (%)	F_{cum} (%)
						0.5	1	1			
			0.5	2	2	1.0	1	2	2	2	2
						1.5	1	3			
0.5	4	4	1.0	2	4	2.0	1	4	4	2	4
						2.5	2	6			
			1.5	4	8	3.0	2	8	6	4	8
						3.5	2	10			
1.0	8	12	2.0	4	12	4.0	2	12	8	4	12
						4.5	3	15			
			2.5	7	19	5.0	4	19	10	7	19
						5.5	6	25			
1.5	58	70	3.0	51	70	6.0	45	70	12	51	70
						6.5	9	79			
			3.5	13	83	7.0	4	83	14	13	83
						7.5	3	86			
2.0	19	89	4.0	6	89	8.0	3	89	16	6	89
						8.5	2	91			
			4.5	4	93	9.0	2	93	18	4	93
						9.5	2	95			
2.5	7	96	5.0	3	96	10.0	1	96	20	3	96
						10.5	1	97			
			5.5	2	98	11.0	1	98	22	2	98
						11.5	1	99			
3.0	4	100	6.0	2	100	12.0	1	100	24	2	100

The longer duration storms tend to be used for sizing detention facilities while at the same time providing a reasonable peak flow for sizing the conveyance system.

ESTIMATION OF EFFECTIVE RAINFALL

Only a fraction of the precipitation which falls during a storm contributes to the overland flow or runoff from the catchment. The balance is diverted in various ways.

Evaporation	In certain climates it is possible that some fraction of the rainfall evaporates before reaching the ground. Since rain fall is measured by gauges on the earth's surface this subtraction is automatically taken into account in recorded storms and may be ignored by the drainage engineer.
Interception	This fraction is trapped in vegetation or roof depressions and never reaches the catchment surface. It is eventually dissipated by evaporation.
Infiltration	Rainfall which reaches a pervious area of the ground surface will initially be used to satisfy the capacity for infiltration in the upper layer of the soil. After even quite a short dry period the infiltration capacity can be quite large (e.g., 100 mm/hr) but this gradually diminishes after the start of rainfall as the storage capacity of the ground is saturated. The infiltrated water will either: a) evaporate directly by capillary rise; b) evapotranspirate through the root system of vegetal cover; c) move laterally through the soil in the form of interflow toward a lake or stream; or, d) penetrate to deeper levels to recharge the ground water.
Surface Depression Storage	If the intensity of the rainfall reaching the ground exceeds the infiltration capacity of the ground, the excess will begin to fill the interstices and small depressions on the ground surface. Clearly this will begin to happen almost immediately on impervious surfaces. Only after these tiny reservoirs have been filled will overland flow commence and contribute to the runoff from the catchment. Since these surface depressions are not uniformly distributed it is quite possible that runoff will commence from some fraction of the catchment area before the depression storage on another fraction is completely filled. Typical recommended values for surface depression storage are given in Table 3.3.

The effective rainfall is thus that portion of the storm rainfall which contributes directly to the surface runoff hydrograph. This might be expressed as follows.

$$\text{Runoff, } Q_t = \text{Precipitation, } P_t - \text{Interception depth} - \text{Infiltrated volume} - \text{Surface Depression Storage}$$

All of the terms are expressed in units of depth.

A number of methods are available to estimate the effective rainfall and thus the amount of runoff for any particular storm event. These range from the runoff coefficient C of the rational method to relatively sophisticated computer implementations of semi-empirical laws representing the physical processes. The method selected should be based on the size of the drainage area, the data available, and the degree of sophistication warranted for the design. Three methods for estimating effective rainfall are outlined.

The Runoff Coefficient C (Rational Method)

If an impervious area, A, is subjected to continuous and long lasting rainfall of intensity i, then after a time (time of concentration T_c) the runoff will be given by the equation:

$$Q = k \cdot i \cdot A$$

The rational method assumes that all of the abstractions may be represented by a single coefficient of volumetric runoff C so that in general the equation reduces to

$$Q = k \cdot C \cdot i \cdot A$$

where: Q = runoff in m^3/s
i = intensity in mm/hr
A = drainage area in hectares
k = constant = 0.00278

When using the rational method, the following assumptions are considered:

a) The rainfall intensity is uniform over the entire watershed during the entire storm duration.

b) The maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration.

c) The time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

Table 3.3 Typical recommended values for surface depression storage [6, 7]

Land Cover	Recommended Value (mm)
Large Paved Areas	2.5
Roofs, Flat	2.5
Fallow Land Field without Crops	5.0
Fields with Crops (grain, root crops)	7.5
Grass Areas in Parks, Lawns	7.5
Wooded Areas and Open Fields	10.0

Since C is the only manipulative factor in the rational formula the runoff is directly proportional to the value assigned to C. Care should be exercised in selecting the value as it incorporates all of the hydrologic abstractions, soil types and antecedant conditions. Table 3.4 lists typical values for C as a function of land use for storms of approximately 5 to 10 year return period. It is important to note that the appropriate value of C depends on the magnitude of the storm and significantly higher values of C may be necessary for more extreme storm events. This is perhaps one of the most serious of the deficiencies associated with this method.

Table 3.4 Recommended runoff coefficients[8]

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighbourhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are:

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tabulations are applicable for storms of 5- to 10-yr frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

The Soil Conservation Service Method

The Soil Conservation Service (SCS) method[3] developed a relationship between rainfall, (P), retention, (S), and effective rainfall or runoff(Q). The retention or potential storage in the soil is established by selecting a curve number (CN). The curve number is a function of soils type, ground cover and Antecedent Moisture Condition (AMC).

The hydrological soil groups, as defined by SCS soil scientists are:

A. (Low runoff potential) Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravel.

B. Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.

C. Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture.

D. (High runoff potential) Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.

Knowing the hydrological soil group and the corresponding land use, the runoff potential or CN value of a site may be determined. Table 3.5 lists typical CN values.

Three levels of Antecedent Moisture Conditions are considered in the SCS method. It is defined as the amount of rainfall in a period of five to thirty days preceding the design storm. In general, the heavier the antecedent rainfall, the greater the runoff potential.

AMC I — Soils are dry but not to the wilting point. This is the lowest runoff potential.

AMC II — The average case.

AMC III — Heavy or light rainfall and low temperatures having occurred during the previous five days. This is the highest runoff potential.

The CN values in Table 3.5 are based on antecedent condition II. Thus, if moisture conditions I or III are chosen, then a corresponding CN value is determined (see Table 3.6).

The potential storage in the soils is based on an initial abstraction (I_a) which is the interception, infiltration and depression storage prior to runoff and infiltration after runoff.

Table 3.5 Runoff curve numbers[2]

Runoff curve number for selected agricultural suburban and urban land use (Antecedent moisture condition II and $I_a = 0.2\ S$)

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP A	B	C	D
Cultivated land[1]:	without conservation treatment	72	81	88	91
	with conservation treatment	62	71	78	81
Pasture or range land:	poor condition	68	79	86	89
	good condition	39	61	74	80
Meadow: good condition		30	58	71	78
Wood or forest land:	thin stand, poor cover, no mulch	45	66	77	83
	good cover[2]	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential: [3]					
Average lot size	Average % Impervious[4]				
1/20 hectare or less	65	77	85	90	92
1/10 hectare	38	61	75	83	87
3/20 hectare	30	57	72	81	86
1/5 hectare	25	54	70	80	85
2/5 hectare	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers[5]		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

[1] For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972[3].
[2] Good cover is protected from grazing and litter and brush cover soil.
[3] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
[4] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
[5] In some warmer climates of the country a curve number of 95 may be used.

Table 3.6 Curve number relationships for different antecedent moisture conditions

CN for Condition II	CN for Conditions I & III		CN for Condition II	CN for Conditions I & III	
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	36	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50	84			
68	48	84	25	12	43
67	47	83	20	9	37
66	46	82	15	6	30
65	45	82	10	4	22
64	44	81	5	2	13
63	43	80	0	0	0
62	42	79			
61	41	78			

The effective rainfall is defined by the relationship.

$$Q = \frac{(P - I_a)^2}{P + S - I_a} \qquad \text{where } S = [(100/CN) - 10]\cdot 25.4$$

The original SCS method assumed the value of I_a to be equal to 0.2 S. However, many engineers have found that this may be overly conservative, especially for moderated rainfall events and low CN values. Under these conditions the I_a value may be reduced to be a lesser percentage of S or may be estimated and input directly to the above equation.

The Horton Infiltration Equation

The Horton equation[9], which defines the infiltration capacity of the soil, changes the initial rate, f_o, to a lower rate, f_c. The infiltration capacity is an upper bound and is realized only when the available rainfall equals or exceeds the infiltration capacity. Therefore, if the infiltration capacity is given by:

$$f_{cap} = f_c + (f_o - f_c)\, e^{-t \cdot k}$$

Then the actual infiltration, f, will be defined by one or the other of the following two equations:

$$f = f_{cap} \text{ for } i \geq f_{cap}$$

$$f = i \quad \text{for } i \leq f_{cap}$$

In the above equations:

f = actual infiltration rate into the soil
f_{cap} = maximum infiltration capacity of the soil
f_o = initial infiltration capacity
f_c = final infiltration capacity
i = rainfall intensity
k = exponential decay constant (1/hours)
t = elapsed time from start of rainfall (hours)

Figure 3.7 shows a typical rainfall distribution and infiltration curve.

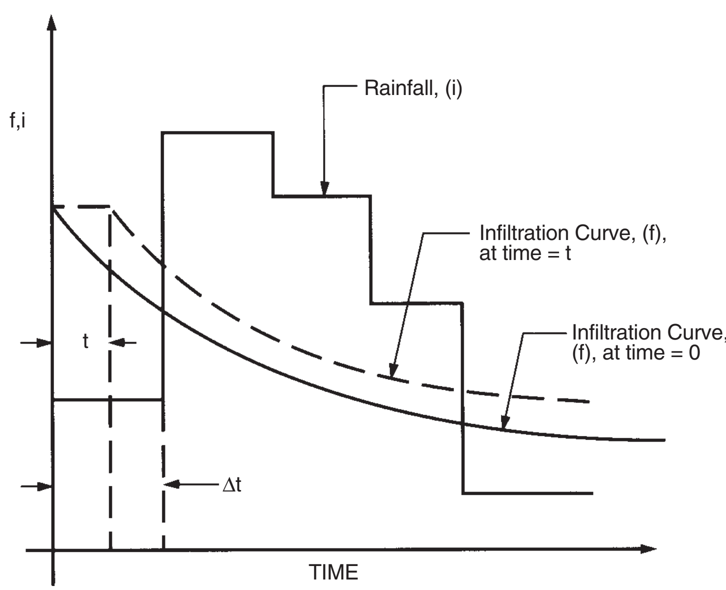

Figure 3.7 Representation of the Horton equation

For the initial timesteps the infiltration rate exceeds the rainfall rate. The reduction in infiltration capacity is dependent more on the reduction in storage capacity in the soil rather than the elapsed time from the start of rainfall. To account for this the infiltration curve should, therefore, be shifted (dashed line for first timestep, Δt) by an elapsed time that would equate the infiltration volume to the volume of runoff.

A further modification is necessary if surface depression is to be accounted for. Since the storage depth must be satisfied before overland flow can occur the initial finite values of the effective rainfall hyetograph must be reduced to zero until a depth equivalent to the surface depression storage has been accumulated. The final hyetograph is the true effective rainfall which will generate runoff from the catchment surface.

The selection of the parameters for the Horton equation depends on soil type, vegetal cover and antecedent moisture conditions. Table 3.7 shows typical values for f_o and f_c (mm/hour) for a variety of soil types under different crop conditions. The value of the lag constant should be typically between 0.04 and 0.08.

Comparison of the SCS and Horton Methods

Figure 3.8 illustrates the various components of the rainfall runoff process for the SCS and Horton methods. The following example serves to show some of the difference between use of the SCS method in which the initial abstraction is used and the moving curve Horton method in which surface depression storage is significant. The incident storm is assumed to be represented by a second quartile Huff curve with a total depth of 50 mm and a duration of 120 minutes. In one case the SCS method is used with the initial abstraction set at an absolute value of I_a = 6.1 mm. The curve number used is 87.6. Figure 3.9(a) shows that no runoff occurs until approximately 30 minutes have elapsed at which time the rainfall has satisfied the initial abstraction. From that point, however, the runoff, although small, is finite and continues to be so right to the end of the storm.

Table 3.7 Typical values for the Horton equation parameters[9]

	Loam, Clay K = 0.08		Clavey Sand K = 0.06		Sand, Loess, Gravel K = 0.04	
Land Surface Types	f_o	f_c	f_o	f_c	f_o	f_c
Fallow land field without crops	15	8	33	10	43	15
Fields with crops (grain, root crops, vines)	36	3	43	8	64	10
Grassed verges, playground, ski slopes	20	3	20	3	20	3
Uncompacted grassy surface, grass areas in parks, lawns	43	8	64	10	89	18
Gardens, meadows, pastures	64	10	71	15	89	18
Coniferous woods	53*	53*	71*	71*	89*	89*
City parks, woodland, orchards	89	53	89	71	89*	89*

*K = 0

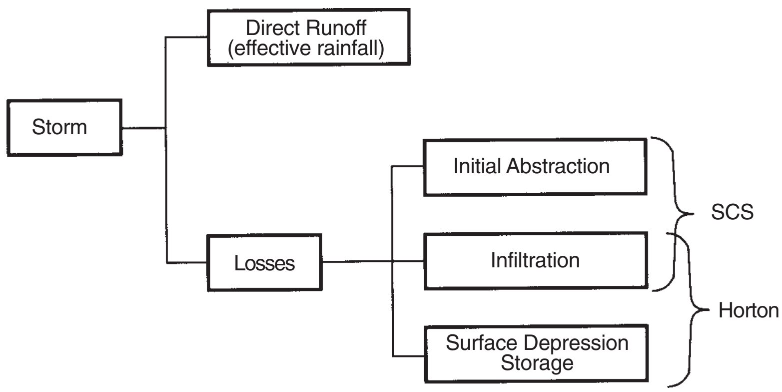

Figure 3.8 Conceptual components of rainfall

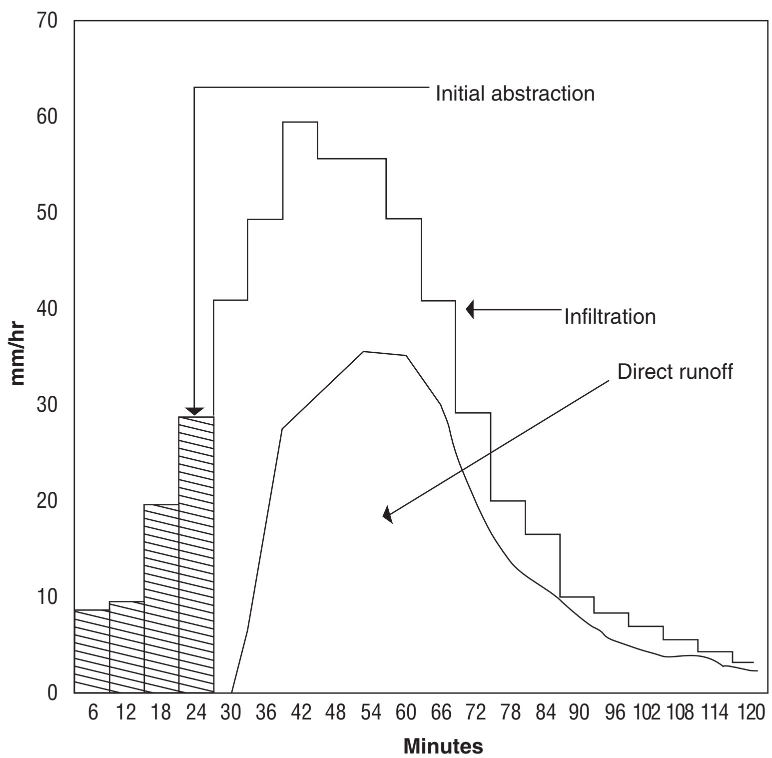

Figure 3.9a SCS Method with I_a = 6.1 mm and CN = 87.6

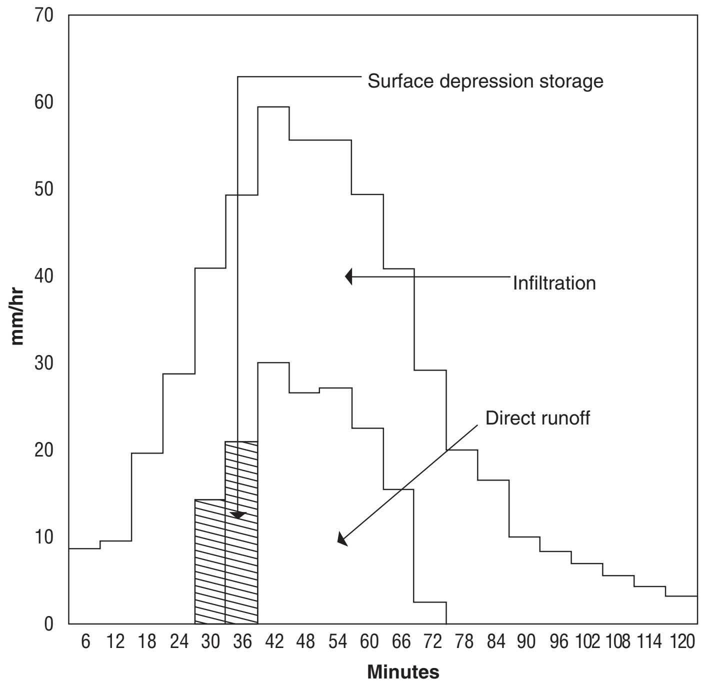

Figure 3.9b Horton equation f_o = 30mm, f_c = 10mm, K = 0.25
Surface depression storage = 4mm

The Horton case is tested using values of f_o = 30 mm/hr; f_c = 10 mm/hr; K = 0.25 hour and a surface depression storage depth of 5 mm.

These values have been found to give the same volumetric runoff coefficient as the SCS parameters. Figure 3.9(b) shows that infiltration commences immediately and absorbs all of the rainfall until approximately 30 minutes have elapsed. However, the initial excess surface water has to fill the surface depression storage which delays the commencement of runoff for a further 13 minutes. Moreover, after 72 minutes the rainfall intensity is less than f_c and runoff is effectively stopped at that time.

lt will be found that the effective rainfall hyetograph generated using the Horton method has more leading and trailing "zero" elements so that the effective hyetograph is shorter but more intense than that produced using the SCS method.

ESTABLISHING THE TIME OF CONCENTRATION

Apart from the area and the percentage of impervious surface, one of the most important characteristics of a catchment is the time which must elapse until the entire area is contributing to runoff at the outflow point. This is generally called the Time of Concentration, T_c. This time is comprised of two components:

(1) The time for overland flow to occur from a point on the perimeter of the catchment to a natural or artificial drainage conduit or channel.
(2) The travel time in the conduit or channel to the outflow point of the catchment.

In storm sewer design the time of concentration may be defined as the inlet time plus travel time. Inlet times used in sewer design generally vary from 5 to 20 minutes, with the channel flow time being determined from pipe flow equations.

Factors Affecting Time of Concentration

The time taken for overland flow to reach a conduit or channel depends on a number of factors:

a) Overland flow length (L). This should be measured along the line of greatest slope from the extremity of the catchment to a drainage conduit or channel. Long lengths result in long travel times.

b) Average surface slope (S). Since T_c is inversely proportional to S care must be exercised in estimating an average value for the surface slope.

c) Surface roughness. In general, rough surfaces result in long travel times and vice versa. Thus, if a Manning equation is used to estimate the velocity of overland flow, T_c will be proportional to the Manning roughness factor, n.

d) Depth of overland flow (y). It seems reasonable to assume that very shallow surface flows move more slowly than deeper flows. However, the depth of flow is not a characteristic of the catchment alone but depends on the intensity of the effective rainfall or surface moisture excess.

Several methods of estimating the Time of Concentration are described below. Since it is clear that this parameter has a strong influence on the shape of the runoff hydrograph, it is desirable to compare the value to that obtained from observation, if possible. In situations were insufficient historical data is available it may help to compare the results obtained by two or more methods. The impact on the resultant hydrograph due to using different methods for establishing the time of concentration should then be assessed.

The Kirpich Formula

This empirical formula[10] relates T_c to the length and average slope of the basin by the equation:

$T_C = 0.00032\, L^{0.77}\, S^{-0.385}$ (See Figure 3.10)

Twin outfall lines for major urban storm sewer system.

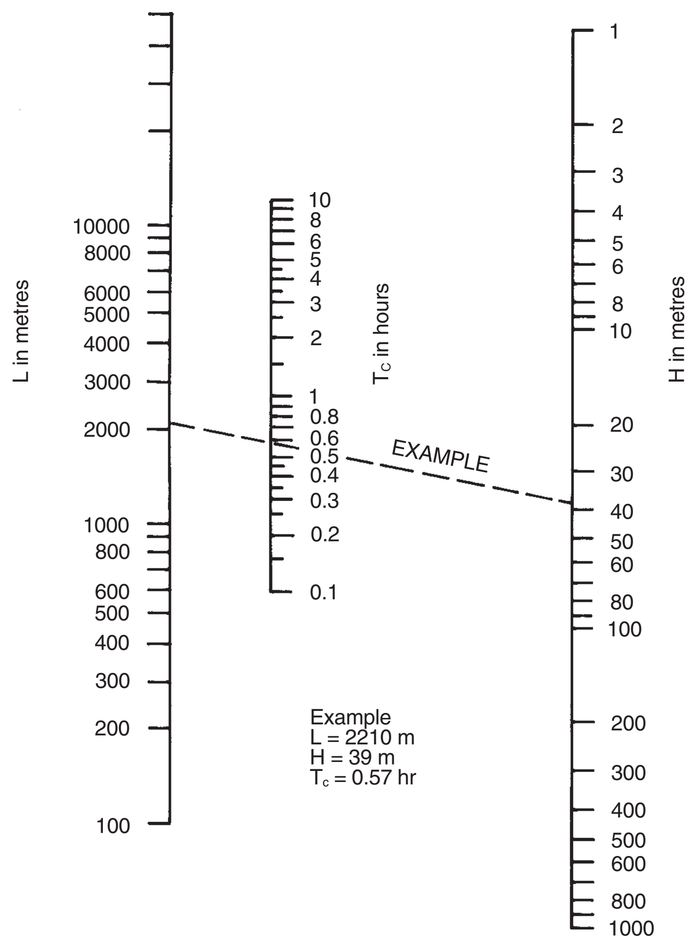

Figure 3.10 T_c nomograph using the Kirpich formula

Where, T_c = time of concentration in hours

L = maximum length of water travel in metres

S = surface slope, given by H/L

H = difference in elevation between the most remote point on the basin and the outlet, in metres

From the definition of L and S it is clear that the Kirpich equation combines both the overland flow or entry time and the travel time on the channel or conduit. It is, therefore, particularly important that in estimating the drop H, the slope S and ultimately the time of concentration T_c, an allowance, if applicable, be made for the inlet travel time.

The Kirpich equation is normally used for natural basins with well defined routes for overland flow along bare earth or mowed grass roadside channels. For overland flow on grassed surfaces the value of T_c obtained should be doubled. For overland flow on concrete or asphalt surfaces the value should be reduced by multiplying by 0.4. For concrete channels, a multiplying factor of 0.2 should be used.

For large watersheds, where the storage capacity of the basin is significant, the Kirpich formula tends to significantly underestimate T_c.

The Uplands Method

When calculating travel times for overland flow in watersheds with a variety of land covers, the Uplands Method[2] may be used. This method relates the time of concentration to the basin slope, length and type of ground cover. The individual times are calculated with their summation giving the total travel time. A graphical solution can be obtained from Figure 3.11. However, it should be noted that the graph is simply a log-log plot of values of $V/S^{0.5}$ given in the following table.

$V/S^{0.5}$ Relationship for Various Land Covers

Land Cover	$V/S^{0.5}$ (m/s)
Forest with heavy ground litter, hay meadow (overland flow)	0.6
Trash fallow or minimum tillage cultivation; contour, strip cropped, woodland (overland flow)	1.5
Short grass pasture (overland flow)	2.3
Cultivated, straight row (overland flow)	2.7
Nearly bare and untilled (overland flow) or alluvial fans in Western mountain regions	3.0
Grassed waterway	4.6
Paved areas (sheet flow); small upland gullies	6.1

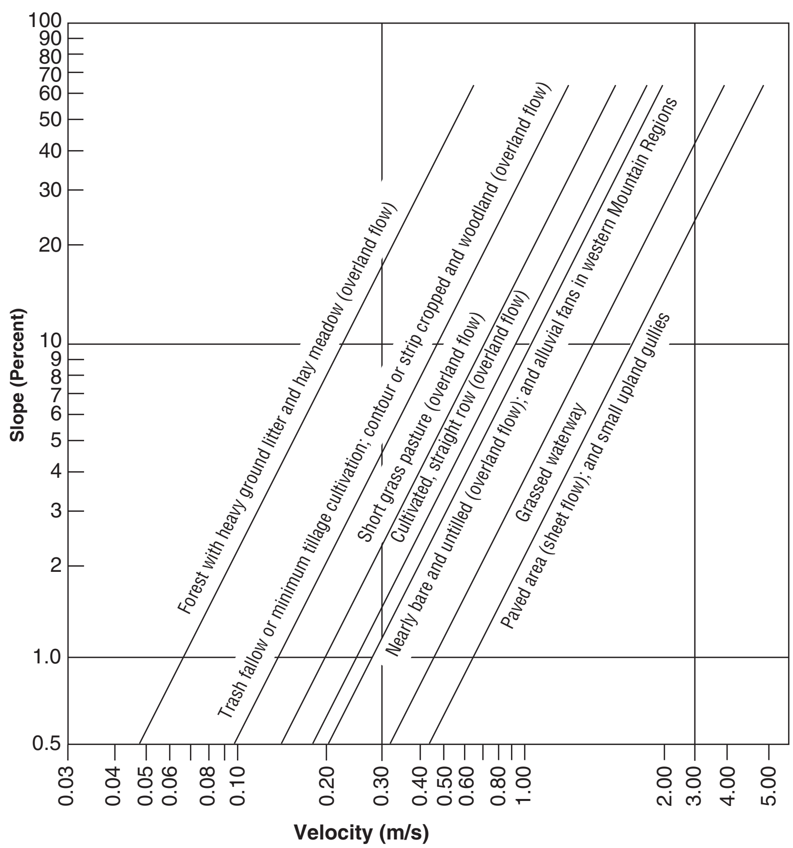

Figure 3.11 Velocities for Upland method for estimating travel time for overland flow

The Kinematic Wave Method

The two methods described above have the advantage of being quite straightforward and may be used for either simple or more complex methods of determining the runoff. Apart from the empirical nature of the equations, the methods assume that the time of concentration is independent of the depth of overland flow or, more generally, the magnitude of the input. A method in common use which is more physically based and which also reflects the dependance of T_c on the intensity of the effective rainfall is the Kinematic Wave method.

The method was proposed by Henderson[12] to analyze the kinematic wave resulting from rainfall of uniform intensity on an impermeable plane surface or rectangular area. The resulting equation is as follows:

$$T_c = k\ (L\ n/\ S)^{0.6}\ i_{eff}^{\ -0.4}$$

in which k = 0.126 for SI units

L = Length of overland flow (m)

n = Manning's roughness coefficient

S = Average slope of overland flow (m/m)

i_{eff} = Effective rainfall intensity (mm/hr)

Other Methods

Other methods have been developed which determine T_c for specific geographic regions or basin types. These methods are often incorporated into an overall procedure for determining the runoff hydrograph. Before using any method the user should ensure that the basis on which the time of concentration is determined is appropriate for the area under consideration.

DETERMINATION OF THE RUNOFF HYDROGRAPH

The following sections outline alternative methods for generating the runoff hydrograph. Emphasis will be given to establishing the hydrograph for single storm events. Methods for estimating flow for urban and rural conditions are given.

Ease of installation of CSP through existing concrete box.

Irrespective of the method used, it should be ensured that wherever possible the results should be compared to historical values. In many cases a calibration/validation exercise will aid in the selection of the most appropriate method.

All of the methods described could be carried out using hand calculations; however, for all but the simplest cases the exercise would be very laborious. Furthermore, access to many tested computer models has been made easier in recent years due to the widespread use of microcomputers. For these reasons emphasis will be placed on describing the basis of each method and the relevant parameters. A subsequent section will relate the methods to several computer models.

Rainfall runoff models may be grouped into two general classifications which are illustrated in Figure 3.12. One approach uses the concept of effective rainfall in which a loss model is assumed which divides the rainfall intensity into losses (i.e., to initial infiltration and depression storage) and effective rainfall. The effective rainfall hyetograph is then used as input to a catchment model to produce a runoff hydrograph. It follows from this approach that infiltration must stop at the end of the storm.

The alternative approach employs a surface water budget in which the infiltration or loss mechanism is incorporated into the catchment model. In this method the storm rainfall is used as input and the estimation of infiltration and other losses is made an integral part of the calculation of runoff. This approach implies that infiltration will continue as long as the average depth of excess water on the surface is finite. Clearly, this may continue after the cessation of rainfall.

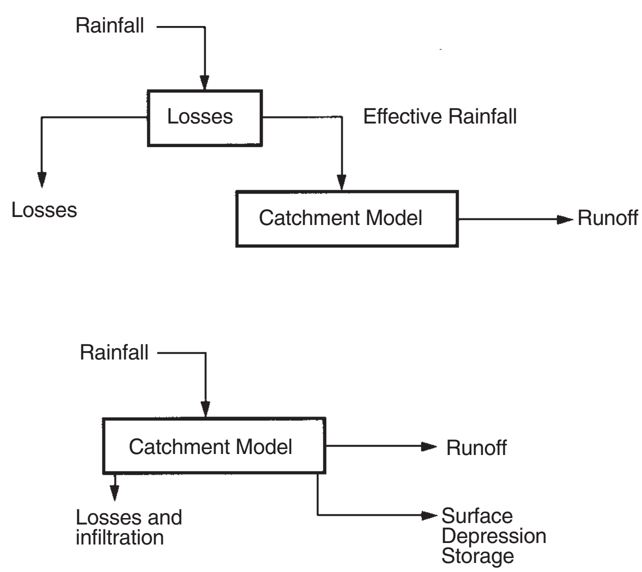

Figure 3.12 Classification of rainfall-runoff models: Effective Rainfall (top) & Surface Water Budget (bottom)

SCS Unit Hydrograph Method

A unit hydrograph represents the runoff distribution over time for one unit of rainfall excess over a drainage area. This method assumes that the ordinates of flow are proportional to the volume of runoff from any storm of the same duration. Therefore, it is possible to derive unit hydrographs for various rainfall blocks by convoluting the unit hydrograph with the effective rainfall distribution. The unit hydrograph theory is based on the following assumptions.

1. For a given watershed, runoff-producing storms of equal duration will produce surface runoff hydrographs with approximately equivalent time bases, regardless of the intensity of the rain.

2. For a given watershed, the magnitude of the ordinates representing the instantaneous discharge from an area will be proportional to the volumes of surface runoff produced by storms of equal duration.

3. For a given watershed, the time distribution of runoff from a given storm period is independent of precipitation from antecedent or subsequent storm periods.

The U.S. Soil Conservation Service, based on the analysis of a large number of hydrographs, proposed a unit hydrograph which requires only an estimate of the time to peak t_p. Two versions of this unit hydrograph were suggested, one being curvilinear in shape, the other being a simple asymmetric triangle as shown in Figure 3.13. In the standard procedure the duration of the recession link is assumed to be $t_r = (^5/_3)\ t_p$ so that the time base is given by $t_b = (^8/_3)t_p$.

The ordinates of the unit hydrograph are expressed in units of discharge per unit depth of effective rainfall. It follows, therefore, that the area under the triangle must equal the total contributing area of the catchment, so that, in terms of the notation used in Figure 3.13:

$$q_p = 2\ A/t_b$$
$$= 0.75\ A/t_p \text{ for } t_b = (\ ^8/_3\ )\ t_p$$

Expressed in SI units the above equation becomes:

$$q_p = 0.75\ (\ A \times 1000^2 \times \tfrac{1}{1000}\ )\ /\ (\ t_p \times 3600\ )$$

or $q_p = 0.208\ A\ /\ t_p$

where A is in km^2
t_p is in hours, and
q_p peak flow is in m^3/s per mm of effective rainfall

The numerical constant in the above equation is a measure of the storage in the watershed. This value, generally denoted as B, is usually taken to be about 0.13 for flat marshy catchments and 0.26 for steep flashy catchments.

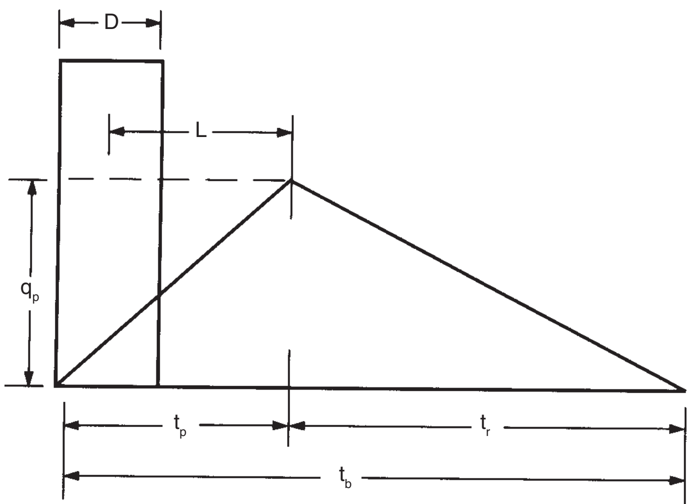

Figure 3.13 SCS triangular unit hydrograph

In Figure 3.13 the definitions of D and L are:

D = excess rainfall period (not to be confused with unit time or unit hydrograph duration)

L = lag of watershed; time of center of mass of excess rainfall (D) to the time to peak (t_p)

The estimate of the time to peak t_p is based on the time of concentration T_c and the time step Δt used in the calculation using the relation:

$$t_p = 0.5\,\Delta t = 0.6\,T_c$$

where T_c may be determined by any acceptable method such as those described in the previous section.

From the above equation it can be seen that the time to peak t_p, and therefore the peak of the Unit Hydrograph q_p, is affected by the value of timestep Δt. Values of Δt in excess of 0.25 t_p should not be used as this can lead to underestimation of the peak runoff.

Rectangular Unit Hydrograph

An alternative option to the triangular distribution used in the SCS method is the rectangular unit hydrograph. Figure 3.14 illustrates the concept of convoluting the effective rainfall with a rectangular unit hydrograph. The ordinate of the unit hydrograph is defined as the area of the unit hydrograph divided by the time of concentration (T_c).

The rational method is often used as a rough estimate of the peak flow. This method, which assumes the peak flow occurs when the entire catchment surface is contributing to runoff, may be simulated using a rectangular unit hydrograph. In this case the effective rainfall hydrograph is reduced to a simple rectangular function and $i_{eff} = k \cdot C \cdot i$. The effective rainfall with duration t_d, is convoluted with a rectangular unit hydrograph which has a base equal to the time of concentration T_c. If t_d is made equal to T_c the resultant runoff hydrograph will be symmetrical and triangular in shape with a peak flow given by $Q = k \cdot C \cdot i \cdot A$ and time base of $t_b = 2\,T_c$. If the rainfall duration t_d is not equal to T_c, then the resultant runoff hydrograph is trapezoidal in shape with peak flow given by the equation below and a time base of $t_b = t_d = T_c$.

$$Q = k \cdot C \cdot i \cdot A\ (t_d / T_c)\ \text{for } t_d \leq T_c$$

and $$Q = k \cdot C \cdot i \cdot A \qquad \text{for } t_d > T_c$$

This approach makes no allowance for the storage effect due to the depth of overland flow and results in an "instantaneous " runoff hydrograph. This may be appropriate for impervious surfaces in which surface depression storage is negligible. However, for pervious or more irregular surfaces it may be necessary to route the instantaneous hydrograph through a hypothetical reservoir in order to more closely represent the runoff hydrograph.

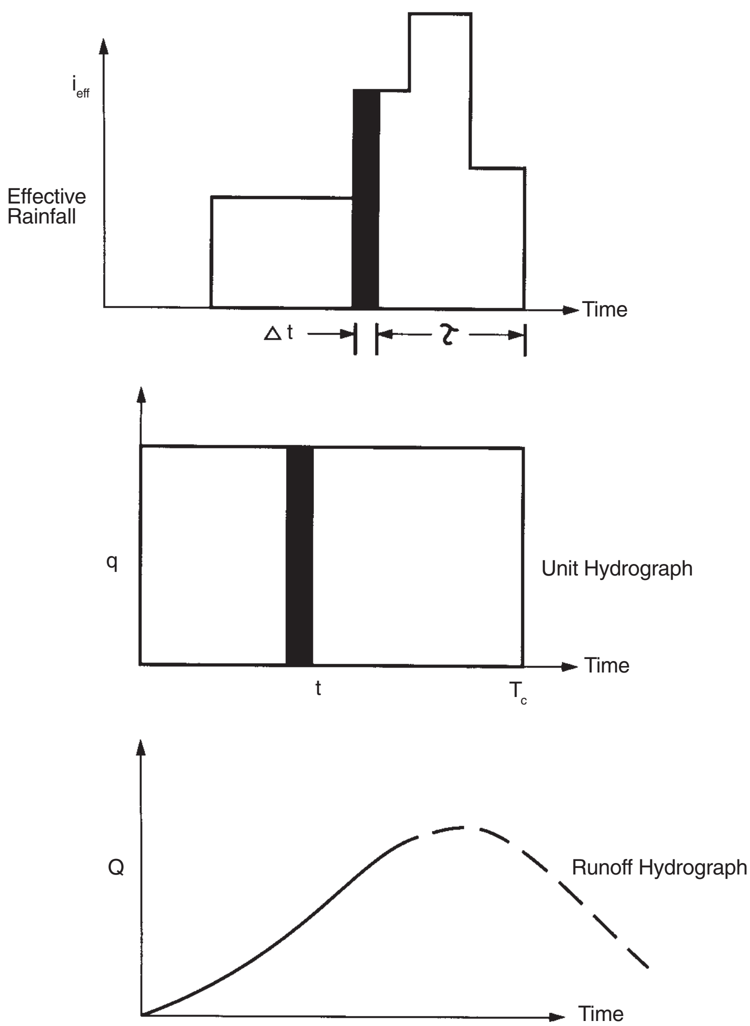

Figure 3.14 Convolution process using a rectangular unit hydrograph

Linear Reservoir Method

A more complex response function was suggested by Pederson[11] in which the shape of the unit hydrograph is assumed to be the same as the response of a single linear reservoir to an inflow of rectangular shape and of duration Δt. A linear reservoir is one in which the storage S is linearly related to the outflow Q by the relation:

$$S = K \cdot Q$$

where K = the reservoir lag or storage coefficient (e.g., in hours)

In the Pederson method the value of K is taken to be 0.5 T_c where T_c is computed from the kinematic wave equation in which the rainfall intensity used is the maximum for the storm being modelled. The use of i_{max} is justified since this intensity tends to dominate the subsequent runoff hydrograph. The resulting Unit Hydrograph is illustrated in Figure 3.15 and comprises a steeply rising limb which reaches a maximum at time t = Δt followed by an exponential recession limb. The two curves can be described by the following equations.

$$q_p = (1\text{-}e^{-\Delta t/k}) / \Delta t \qquad \text{at } t = \Delta t$$

and $$q = q_p \bullet e^{-(t - \Delta t)/k} \qquad \text{for } t > \Delta t$$

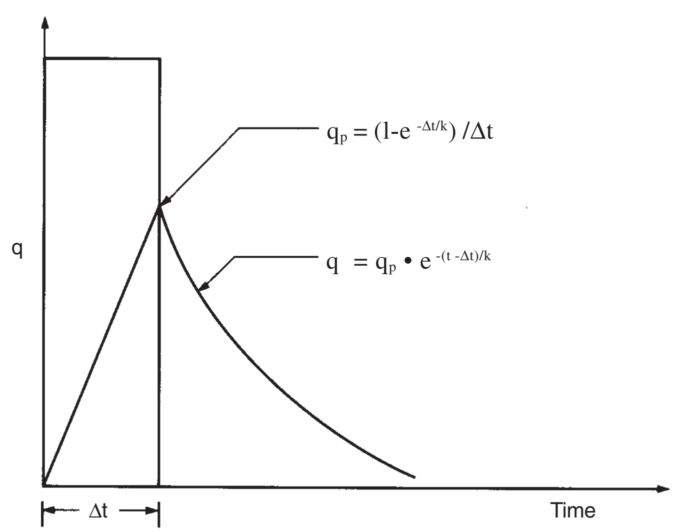

Figure 3.15 The single linear reservoir

An important feature of the method is that the unit hydrograph always has a time to peak at Δt and is incapable of reflecting different response times as a function of catchment length, slope or roughness. It follows that the peak of the runoff hydrograph will usually be close to the time of peak rainfall intensity irrespective of the catchment characteristics.

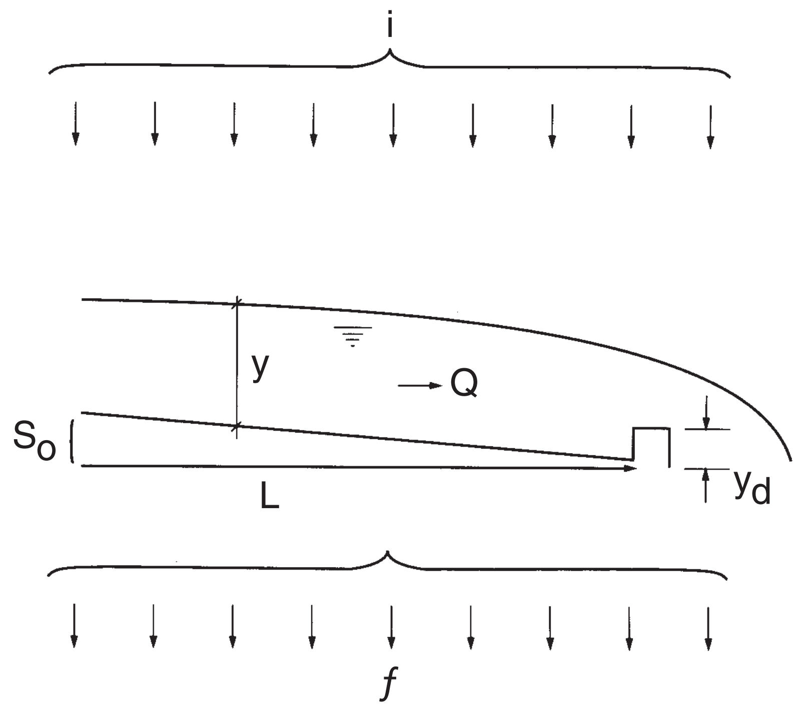

Figure 3.16 Representation of the SWMM/Runoff algorithm

SWMM Runoff Algorithm

The Storm Water Management Model was originally developed jointly for the U.S. Environmental Protection Agency in 1971.[13] Since then it has been expanded and improved by EPA and many other agencies and companies. In particular, the capability for continuous simulation has been added to single event simulation, quality as well as quantity is simulated and snowmelt routines are included in some versions.

The model is intended for use in urban or partly urbanized catchments. It comprises five main "blocks" of code in addition to an Executive Block or supervisory calling program. This section describes the basic algorithm of the Runoff Block which is used to generate the runoff hydrograph in the drainage system, based on a rainfall hyetograph, antecedent moisture conditions, land use and topography.

The method differs from those described above in that it does not use the concept of effective rainfall, but employs a surface water budget approach in which rainfall, infiltration, depression storage and runoff are all considered as processes occurring simultaneously at the land surface. The interaction of these inputs and outputs may be visualized with reference to Figure 3.16.

Treating each sub-catchment as an idealized, rectangular plane surface of breadth B and length L, the continuity or mass balance equation at the land surface is given by the statement:

Inflow = (Infiltration + Outflow) + Rate of Surface Ponding

That is:

$$i \cdot L \cdot B = (f \cdot L \cdot B + Q) + L \cdot B \cdot (\Delta y / \Delta t)$$

where i = Rainfall intensity
f = Infiltration rate
Q = Outflow
y = Depth of flow over the entire surface

The depth of flow (y) is computed using the Manning equation, taking into account the depth of depression surface storage (y_d) which is also assumed to be uniform over the entire surface. This is the dynamic equation.

$$Q = B\ (1/n)\ (y - y_d)^{5/3}\ S^{1/2}$$

where n = Manning's roughness coefficient for overland flow
S = Average slope of the overland flow surface

The infiltration rate (f) must be computed using a method such as the "moving curve" Horton equation or the Green-Ampt model. Infiltration is assumed to occur as long as excess surface moisture is available from rainfall, depression storage or finite overland flow.

It is important to note that the value of Manning's "n" used for overland flow is somewhat artificial (e.g., in the range 0.1 to 0.4) and does not represent a value which might be used for channel flow calculation.

Various methods can be used for the simultaneous solution of the continuity and dynamic equation. One method is to combine the equations into one nondifferential equation in which the depth (y) is the unknown. Once y is determined (e.g., by an interactive scheme such as the Newton-Raphson method) the outflow Q follows.

COMPUTER MODELS

In recent years, many computer models have been developed for the simulation of the rainfall/runoff process. Table 3.8 lists several of these models and their capabilities.

Saddle branch manhole is bolted in place.

Table 3.8 Hydrologic computer models

Model Characteristics	HEC-1[14]	HYMO[15]	HSPF[16]	ILLUDAS[17]	MIDUSS[18]	OTTHYMO[19]	QUALHYMO[20]	SCS TR-20[21]	SCS TR-55[21]	SSARR[22]	STANFORD[23]	STORM[24]	SWMM[13]	USDAHL-74[25]
Model Type:														
Single Event	●	●		●	●	●		●	●				●	
Continuous			●				●			●	●	●	●	●
Model Components:														
Infiltration	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Evapotranspiration			●							●	●	●	●	●
Snowmelt	●		●							●	●	●	●	●
Surface Runoff	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Subsurface Flow	●		●				●			●	●			●
Reservoir Routing	●	●	●		●	●	●	●	●	●			●	
Channel Routing	●	●	●	●	●	●	●	●	●	●	●		●	
Water Quality			●				●					●	●	
Application:														
Urban Land Use			●	●	●	●	●		●			●	●	
Rural Land Use	●	●	●		●	●	●	●		●	●	●		●
Ease of Use:														
High	●	●		●	●	●	●	●	●			●		●
Low			●							●	●		●	

Laying Full Bituminous coated and Full Paved CSP.

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